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EFFECTS OF FOUNDATION EMBEDMENT DURING BUILDING–SOIL INTERACTION

JAVIER AVILÉS[1,*] AND LUIS E. PÉREZ-ROCHA[2]

[1] *Instituto Mexicano de Tecnología del Agua, Jiutepec 62550, Morelos, México*
[2] *Centro de Investigación Sísmica, Carretera al Ajusco 203, Padierna 14200, México*

SUMMARY

A numerical solution for evaluating the effects of foundation embedment on the effective period and damping and the response of soil–structure systems is presented. A simple system similar to that used in practice to account for inertial interaction effects is investigated, with the inclusion of kinematic interaction effects for the important special case of vertically incident shear waves. The effective period and damping are obtained by establishing an equivalence between the interacting system excited by the foundation input motion and a replacement oscillator excited by the free-field ground motion. In this way, the use of standard free-field response spectra applicable to the effective period and damping of the system is permitted. Also, an approximate solution for total soil–structure interaction is presented, which indicates that the system period is insensitive to kinematic interaction and the system damping may be expressed as that for inertial interaction but modified by a factor due to kinematic interaction. Results involving both kinematic and inertial effects are compared with those obtained for no soil–structure interaction and inertial interaction only. The more important parameters involved are identified and their influences are examined over practical ranges of interest. © 1998 John Wiley & Sons, Ltd.

KEY WORDS: foundation embedment; soil–structure interaction; inertial interaction; kinematic interaction; system period; system damping

INTRODUCTION

It is well known that soil–structure interaction produces kinematic and inertial effects. As a result, the motion experienced by the foundation is different from the free-field ground motion. The exact analysis of interaction may be implemented in two steps. First, the foundation input motion is calculated, which involves a reduction of the translational response of the foundation and the generation of rocking and torsional response components. Next, the building–soil system is analysed using as base excitation the foundation input motion.

The effects of soil–structure interaction have been the subject of previous studies[1–4] showing that they may be approximated using the free-field ground motion as the foundation input motion and modifying the fundamental period and associated damping of the structure. Even though this approach excludes the kinematic interaction effects, it has been implemented in major building codes[5,6] because it allows the direct use of standard response spectra for fixed-base systems. Nevertheless, when the effects of kinematic interaction are important, this approach would require that the structural response be evaluated for the foundation input motion rather than for the free-field ground motion.[7,8]

* Correspondence to: Javier Avilés, Instituto Mexicano de Tecnología del Agua, IMTALAB-Geotecnica, Paseo Cuauhnahuac 8532, Col. Progresso, Jiutepec 62550, Morelos, México. E-mail: javiles@tajin.imta.mx

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The effective period and damping of soil–structure systems have been extensively studied either for surface-supported foundations[1–4,7,9–12] or for embedded foundations.[13,14] However, they have been examined at the exclusion of the kinematic interaction, utilizing as base excitation a purely horizontal harmonic motion with constant amplitude. These solutions overestimate the translational component and neglect the rotational components of the foundation input motion. The effects of kinematic interaction on the effective period and damping and the structural response due to the depth of embedment are not yet well understood. By using a simplified two-dimensional model, Todorovska and Trifunac[15] have shown that the system period practically does not depend on the type of incident waves and their angle of incidence, and that the system damping is usually underestimated when kinematic interaction effects are excluded. Based on the same model, Todorovska[16] has found that the system damping is generally larger when the foundation depth is smaller. This conclusion is in contradiction with the results obtained for inertial interaction only.[13,14]

The aim of this work is to evaluate the effects of foundation embedment on the effective period and damping and the response of soil–structure systems, considering both kinematic and inertial interaction. A numerical solution is presented for the simple system formed by a one-storey structure supported by a rigid square foundation embedded in a homogeneous elastic half-space. The system period and system damping are obtained by establishing an analogy between the interacting system subjected to the foundation input motion and a replacement oscillator subjected to the free-field ground motion. The effective period and damping are taken such that the magnitude and location of the resonant response to harmonic excitation are identical for the actual system and the equivalent oscillator. The practical advantage of this approach is that the peak structural response can be determined from standard response spectra for the specified free-field ground motion, in combination with the modified dynamic properties of the fixed-base structure. By introducing some simplifying assumptions, an approximate solution for total soil–structure interaction is also presented; its accuracy is assessed through comparisons with the numerical solution. It is shown that the system damping may be expressed as that for inertial interaction but modified by a factor due to kinematic interaction, which depends on the translational and rocking components of the foundation input motion and on the structure height and foundation depth; the system period is insensitive to kinematic interaction.

We will examine the effects of foundation embedment for vertically propagating plane shear waves only. Because of the characteristics of this wave excitation, the torsional component of the foundation input motion is neglected. Fundamental steps in the analysis of interaction are the evaluations of the impedance functions and input motions of the foundation. The harmonic response of the soil–structure system is computed making use of the complex-valued impedance functions and input motions presented in tabular form by Mita and Luco.[17] The effective period and damping are measured directly from the harmonic response spectrum of the interacting system, in which both inertial and kinematic interaction effects are included. Results for total soil–structure interaction are compared with those obtained for no soil–structure interaction and inertial interaction only. The information and concepts developed for embedded foundations are related to those existing for surface-supported foundations.

SOIL–STRUCTURE SYSTEM

The soil–structure system considered is shown in Figure 1. This model is appropriate to account for interaction effects on the fundamental mode of multistorey structures that respond as a single oscillator in their fixed-base condition. The building is characterized by the fundamental period, T, associated damping, ζ, effective mass, M, and effective height, H, of the fixed-base structure; the rotational inertia of the structural mass is neglected. The foundation is idealized as a rigid square mat of half-width L, depth D, mass M_0 and mass moment of inertia J_0 about a centroidal axis at the base. Both M and M_0 are assumed to be uniformly distributed over identical square areas; the latter is presumed to be located at half the depth of embedment.

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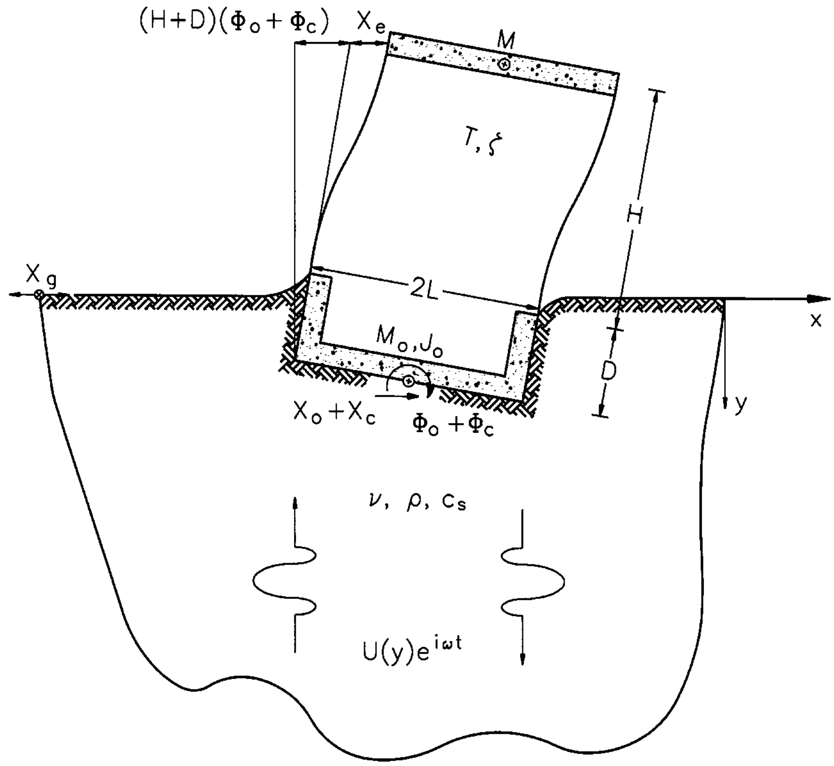

Figure 1. Single structure with square foundation embedded in a homogeneous half-space under vertically incident shear waves

The supporting medium is idealized as a uniform elastic half-space characterized by the Poisson's ratio, ν, mass density, ρ, and shear wave velocity, c_s, of the soil. Welded contact conditions between the foundation and surrounding soil are assumed, so that no uplifting or sliding can take place. The interacting system has three degrees of freedom defined by the deformation of the structure, X_e, and the translation and rocking of the foundation, X_c and Φ_c, relative to the foundation input motion.

The seismic excitation is given by harmonic shear waves propagating vertically, with particle motion along the x-axis. So the free-field ground motion that would exist in the absence of the foundation is

$$U = X_g \cos\left(\frac{\omega y}{c_s}\right) \tag{1}$$

where X_g is the amplitude of the free-field motion at the ground surface and ω the exciting frequency; the factor $e^{i\omega t}$ for the time dependency of harmonic waves has been omitted. This wave excitation induces a foundation input motion with translational and rocking components denoted by the amplitudes X_0 and Φ_0, respectively.

Equilibrium equations

For small vibrations, the lateral displacement of the structure is equal to $X_0 + X_c + (H + D)(\Phi_0 + \Phi_c) + X_e$, whereas the horizontal displacement and vertical rotation of the foundation are equal to $X_0 + X_c$ and $\Phi_0 + \Phi_c$, respectively. The equilibrium of forces acting on the building is given by

$$-\omega^2 M(X_c + (H + D)\Phi_c + X_e) + (i\omega C + K)X_e = \omega^2 M(X_0 + (H + D)\Phi_0) \tag{2}$$

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while the equilibrium of forces acting on the foundation and the equilibrium of moments about the centre of the base are given by

$$
\begin{aligned}
&-\omega^2 M_0\left(X_c+\frac{D}{2}\Phi_c\right)+(\mathrm{i}\omega C_{\mathrm{h}}+K_{\mathrm{h}})X_c+(\mathrm{i}\omega C_{\mathrm{hr}}+K_{\mathrm{hr}})\Phi_c-F_0\\
&\quad=\omega^2 M_0\left(X_0+\frac{D}{2}\Phi_0\right)
\end{aligned}
\tag{3}
$$

$$
\begin{aligned}
&-\omega^2\left(M_0\frac{D}{2}X_c+J_0\Phi_c\right)+(\mathrm{i}\omega C_{\mathrm{hr}}+K_{\mathrm{hr}})X_c+(\mathrm{i}\omega C_{\mathrm{r}}+K_{\mathrm{r}})\Phi_c-M_0\\
&\quad=\omega^2\left(M_0\frac{D}{2}X_0+J_0\Phi_0\right)
\end{aligned}
\tag{4}
$$

where $K=4\pi^2M/T^2$ and $C=4\pi\zeta M/T$ are the linear stiffness and viscous damping of the fixed-base structure, respectively; $F_0=(\mathrm{i}\omega C+K)X_e$ and $M_0=F_0(H+D)$ are the force and moment, respectively, that the structure exerts on the soil. The supporting medium has been replaced with the linear springs K_{h}, K_{r} and K_{hr} and the viscous dashpots C_{h}, C_{r} and C_{hr} for the horizontal, rocking and coupling modes of vibration. The frequency-dependent values used herein are taken from tables reported by Mita and Luco[17] for the soil material dampings $\zeta_{\mathrm{S}}=0{\cdot}001$ and $\zeta_{\mathrm{P}}=0{\cdot}0005$, where ζ_{S} and ζ_{P} represent the hysteretic damping ratios for S- and P-waves, respectively,

Expressing F_0 and M_0 in terms of equation (2) and substituting into equations (3) and (4), respectively, the equilibrium equations of the building–foundation system can be written in matrix form as

$$[\mathbf{K}+\mathrm{i}\omega\mathbf{C}-\omega^2\mathbf{M}]\{\mathbf{X}\}=\omega^2X_{\mathrm{g}}\{Q_{\mathrm{h}}\mathbf{M}_0+Q_{\mathrm{r}}\mathbf{J}_0\} \tag{5}$$

where $\mathbf{X}=\{X_e, X_c, \Phi_c\}^{\mathrm{T}}$ is the vector of displacement amplitudes of the system; $\mathbf{M}_0$ and $\mathbf{J}_0$ are load vectors defined by

$$\mathbf{M}_0=\begin{Bmatrix} M \\ M+M_0 \\ M(H+D)+M_0D/2 \end{Bmatrix} \tag{6}$$

$$\mathbf{J}_0=\begin{Bmatrix} M(H+D) \\ M(H+D)+M_0D/2 \\ M(H+D)^2+J_0 \end{Bmatrix} \tag{7}$$

whereas $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ are the mass, damping and stiffness matrices of the system, respectively, defined by

$$\mathbf{M}=\begin{bmatrix} M & M & M(H+D) \\ M & M+M_0 & M(H+D)+M_0D/2 \\ M(H+D) & M(H+D)+M_0D/2 & M(H+D)^2+J_0 \end{bmatrix} \tag{8}$$

$$\mathbf{C}=\begin{bmatrix} C & 0 & 0 \\ 0 & C_{\mathrm{h}} & C_{\mathrm{hr}} \\ 0 & C_{\mathrm{hr}} & C_{\mathrm{r}} \end{bmatrix} \tag{9}$$

$$\mathbf{K}=\begin{bmatrix} K & 0 & 0 \\ 0 & K_{\mathrm{h}} & K_{\mathrm{hr}} \\ 0 & K_{\mathrm{hr}} & K_{\mathrm{r}} \end{bmatrix} \tag{10}$$

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The ratios $Q_h = X_0/X_g$ and $Q_r = \Phi_0/X_g$ represent the transfer functions of the components of the foundation input motion. They relate the amplitudes of the translational and rocking input motions, respectively, to the amplitude of the free-field motion at the ground surface. The frequency-dependent values used herein are given in a latter section. It should be noted that the kinematic interaction is excluded by setting $Q_h = 1$ and $Q_r = 0$, with which the governing equation for the solely inertial interaction is recovered.[13]

System parameters

The dimensionless parameters that can be used conveniently to evaluate building–soil interaction have been previously identified for surface-supported structures.[3,4] In order of importance and adapted to square foundations, they are:

(1) The wave parameter $\tau_H = H/(c_s T)$, which is a measure of the relative stiffness of the structure and soil. It is useful in evaluating interaction effects for harmonic excitation. For transient excitation, however, the wave parameter $\tau_L = L/(c_s T_0)$ is preferable, T_0 being a characteristic period of the excitation. Both parameters are related by $\tau_L = \tau_H (L/H)(T/T_0)$.
(2) The ratio H/L of the structure height to the half-width of the foundation.
(3) The ratio T/T_0 of the fundamental period of the fixed-base structure to the exciting period.
(4) The relative mass density for the structure and soil, $M/(4\rho L^2 H)$.
(5) The ratio M_0/M of the foundation mass to the mass of the structure.

The following two additional parameters are required for embedded foundations, the first of which is the most important.

(1) The ratio D/L of the foundation embedment to the half-width of the foundation. Alternatively, the wave parameter $\tau_D = D/(c_s T_0)$ may be used. Expressed in terms of τ_L and D/L, it takes the form $\tau_D = \tau_L D/L$.
(2) The mass moment of inertia ratio of the foundation and structure, $J_0/(M(H+D)^2)$.

For the results reported herein, it is assumed that $M/(4\rho L^2 H) = 0{\cdot}15$, $M_0/M = 0{\cdot}25$ and $J_0/(M(H+D)^2) = 0{\cdot}05$. Also, the structural damping and Poisson's ratio are taken as $\zeta = 0{\cdot}05$ and $\nu = \frac{1}{3}$, respectively. These values are representative for typical buildings and soils.

SYSTEM PERIOD AND DAMPING

The interaction procedure stipulated in current seismic codes[5,6] can be adapted to account for the effects of kinematic interaction. It is only necessary to replace the free-field ground motion by the foundation input motion, and the response of the modified structure to this base excitation is then evaluated.[7,8] However, to avoid the inconvenience of dealing with the foundation input motion, it would be highly desirable that both inertial and kinematic interaction effects are included by merely modifying the fundamental period and associated damping of the structure. In this manner, the resulting modified structure may be analysed using the free-field ground motion as the base excitation.

The kinematic interaction effects may be simply accounted for by establishing an analogy between the interacting system subjected to the foundation input motion and a replacement oscillator subjected to the free-field ground motion, as illustrated in Figure 2. The effective period and damping are determined so that, under harmonic excitation, the resonant period and the peak pseudo-acceleration of the replacement oscillator are equal to those of the interacting system.

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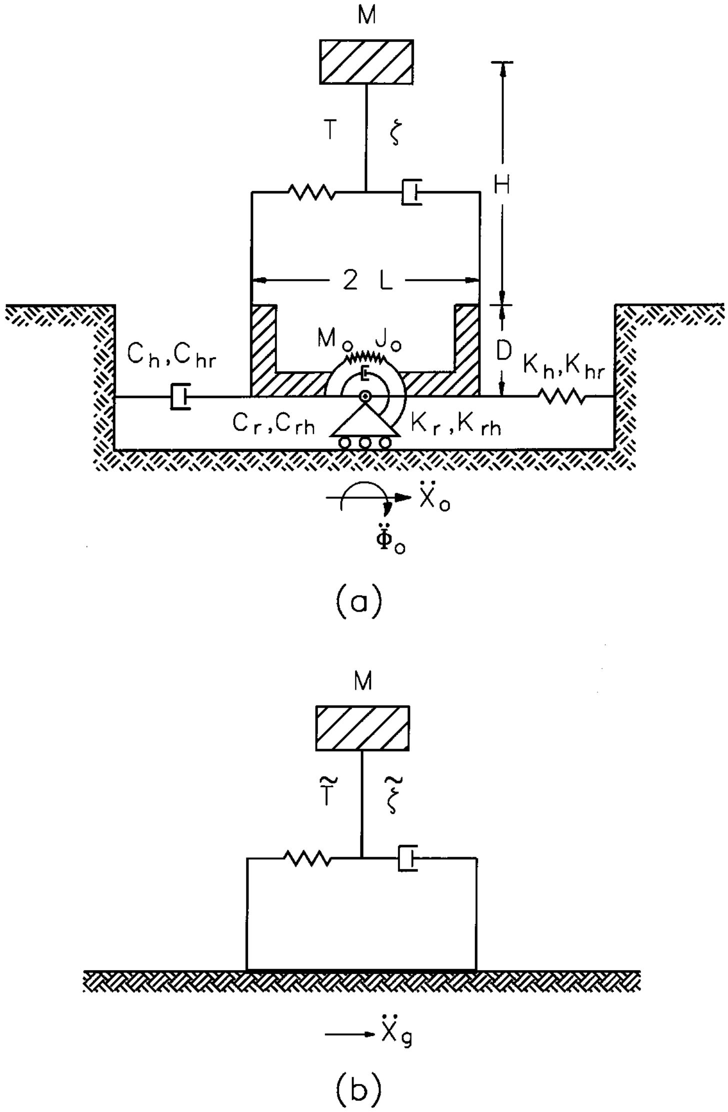

Figure 2. (a) Soil–structure system subjected to the foundation input motion; and (b) replacement oscillator subjected to the free-field ground motion

If the transfer function $Q = \Omega^2 X_e / \omega^2 X_g$ of the soil–structure system is known, $\Omega = 2\pi/T$ being the fundamental frequency of the fixed-base structure, then the overall period and damping of the system, $\tilde{T}$ and $\tilde{\zeta}$, are computed from the well-known expressions[4]

$$\tilde{T} = \sqrt{1 - 2\tilde{\zeta}^2}\, T_m \tag{11}$$

$$Q_m = \frac{1}{2\tilde{\zeta}\sqrt{1 - \tilde{\zeta}^2}} \tag{12}$$

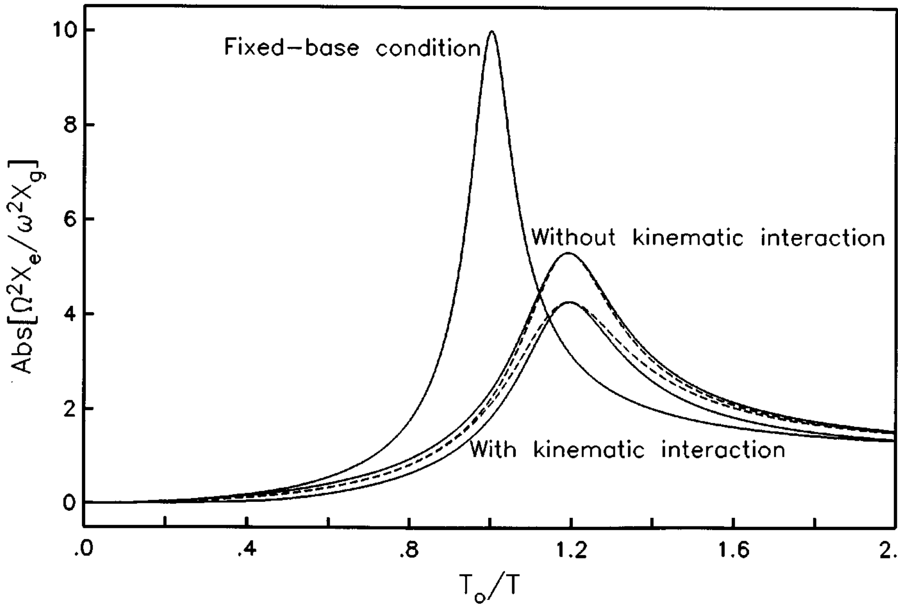

Figure 3. Harmonic response spectra for an interacting system with $D/L = 1$, $H/L = 1$ and $\tau_H = 0.2$; actual system (solid line) and replacement oscillator (dashed line)

where Q_m is the absolute maximum value of the pseudo-acceleration and T_m the corresponding resonant period measured at the transfer function of the system. After some manipulations, the system damping results in

$$\zeta = \frac{1}{\sqrt{2}}\left(1 - \sqrt{\frac{Q_m^2 - 1}{Q_m^2}}\right)^{1/2} \tag{13}$$

With the effective period and damping determined by this way, an excellent agreement between the harmonic response spectra of the interacting system and the replacement oscillator is obtained, as shown in Figure 3 for $D/L = 1$, $H/L = 1$ and $\tau_H = 0{\cdot}2$. The abscissas represent the ratio T_0/T, where $T_0 = 2\pi/\omega$ is the period of the exciting motion. Results are displayed for three cases: (1) the fixed-base condition; (2) without kinematic interaction; and (3) with kinematic interaction. It can be seen that the resonant periods are almost identical whether or not the kinematic interaction is considered. The difference in the peak responses for cases (2) and (3) reflects the effects of kinematic interaction on the system damping.

This procedure may be viewed as an extension of the replacement oscillator approach used to account for the inertial interaction effects.[2–4] It is quite convenient for design purposes, since both inertial and kinematic effects are expressed by an increase in the fundamental period and a change in the associated damping of the fixed-base structure. Thus, standard free-field response spectra can be used to assess the peak structural response.

APPROXIMATION TO KINEMATIC INTERACTION

Due to the lack of simplified solutions, the kinematic interaction effects have not been incorporated so far in seismic design provisions for building structures. It is possible to deduce a practical criterion by ignoring the

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mass M_0 and the mass moment of inertia J_0 of the foundation, as well as the coupled stiffness K_{hr} and damping C_{hr} of the soil in comparison with the translational and rocking terms. Such assumptions have been extensively used by many authors[1,9–13] to derive approximate expressions for the overall period and damping of soil–structure systems involving only the inertia interaction effects. Introducing these simplifications, equation (5) takes the reduced form

$$\left[\begin{bmatrix} K & 0 & 0 \\ 0 & K_h & 0 \\ 0 & 0 & K_r \end{bmatrix} + i\omega \begin{bmatrix} C & 0 & 0 \\ 0 & C_h & 0 \\ 0 & 0 & C_r \end{bmatrix} - \omega^2 \begin{bmatrix} M & M & M(H+D) \\ M & M & M(H+D) \\ M(H+D) & M(H+D) & M(H+D)^2 \end{bmatrix}\right] \begin{Bmatrix} X_e \\ X_c \\ \Phi_c \end{Bmatrix}$$

$$= \omega^2 X_g \left\{ Q_h \begin{Bmatrix} M \\ M \\ M(H+D) \end{Bmatrix} + Q_r \begin{Bmatrix} M(H+D) \\ M(H+D) \\ M(H+D)^2 \end{Bmatrix} \right\} \tag{14}$$

Dividing the two first rows of equation (14) by $\omega^2 M$ and the last one by $\omega^2 M(H+D)$, this equation reduces to

$$\begin{bmatrix} \frac{\Omega^2}{\omega^2}(1+i2\zeta')-1 & -1 & -1 \\ -1 & \frac{\Omega_h^2}{\omega^2}(1+i2\zeta_h')-1 & -1 \\ -1 & -1 & \frac{\Omega_r^2}{\omega^2}(1+i2\zeta_r')-1 \end{bmatrix} \begin{Bmatrix} X_e \\ X_c \\ (H+D)\Phi_c \end{Bmatrix}$$

$$= X_g(Q_h + (H+D)Q_r) \begin{Bmatrix} 1 \\ 1 \\ 1 \end{Bmatrix} \tag{15}$$

where Ω_h and Ω_r are the natural frequencies for the translation and rocking of the rigid-assumed structure, respectively, which are defined as

$$\Omega_h^2 = \frac{K_h}{M} \tag{16}$$

$$\Omega_r^2 = \frac{K_r}{M(H+D)^2} \tag{17}$$

while $\zeta' = (\omega/\Omega)\zeta$, $\zeta_h' = (\omega/\Omega_h)\zeta_h$ and $\zeta_r' = (\omega/\Omega_r)\zeta_r$, in which ζ_h and ζ_r are the damping ratios of the soil for the translational and rocking modes of the foundation, respectively. They are given by

$$\zeta_h = \frac{C_h}{2\Omega_h M} \tag{18}$$

$$\zeta_r = \frac{C_r}{2\Omega_r M(H+D)^2} \tag{19}$$

By solving the complex system of algebraic equations given in equation (15), neglecting all second-order damping terms, the normalized pseudo-acceleration of the interacting system results in

$$\frac{\Omega^2 X_e}{\omega^2 X_g} = (Q_h + (H+D)Q_r)\left(1 - \frac{\omega^2}{\Omega^2} - \frac{\omega^2}{\Omega_h^2} - \frac{\omega^2}{\Omega_r^2} + i2\left(\zeta' - \frac{\omega^2}{\Omega_h^2}(\zeta' - \zeta_h') - \frac{\omega^2}{\Omega_r^2}(\zeta' - \zeta_r')\right)\right)^{-1} \tag{20}$$

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The system frequency $\tilde{\Omega}$ is determined from the resonance condition that requires the undamped response be infinite at $\omega = \tilde{\Omega}$. It is apparent from equation (20) that, setting $\zeta' = \zeta'_h = \zeta'_r = 0$, this resonant frequency proves to be

$$\frac{1}{\tilde{\Omega}^2} = \frac{1}{\Omega^2} + \frac{1}{\Omega_h^2} + \frac{1}{\Omega_r^2} \tag{21}$$

To determine the system damping $\tilde{\zeta}$, the absolute maximum value of the pseudo-acceleration of the interacting system, obtained from equation (20) by substituting $\omega = \tilde{\Omega}$, is equated to $\frac{1}{2}\tilde{\zeta}$ corresponding to the resonant response of the replacement oscillator. This leads to

$$\tilde{\zeta} = |Q_h + (H + D)Q_r|^{-1}\left(\zeta\frac{\tilde{\Omega}^3}{\Omega^3} + \zeta_h\frac{\tilde{\Omega}^3}{\Omega_h^3} + \zeta_r\frac{\tilde{\Omega}^3}{\Omega_r^3}\right) \tag{22}$$

where the transfer functions Q_h and Q_r, the damping ratios ζ_h and ζ_r, and the natural frequencies Ω_h and Ω_r are evaluated at $\omega = \tilde{\Omega}$.

Equations (21) and (22) are similar to those derived by several authors[1,9–13] for the system frequency and system damping when the effects of kinematic interaction are excluded, except that the latter is divided by the factor $|Q_h + (H + D)Q_r|$ that represents the contribution of the kinematic interaction to the energy dissipation in the interacting system. Accordingly, the effective period and damping of the system with both kinematic and inertial interaction can be estimated as

$$\tilde{T}_k = \tilde{T}_i \tag{23}$$

$$\tilde{\zeta}_k = \frac{\tilde{\zeta}_i}{|Q_h + (H + D)Q_r|} \tag{24}$$

where $\tilde{T}_i$ and $\tilde{\zeta}_i$ are the system period and system damping for inertial interaction only. It is evident from equation (24) that the system damping for total soil–structure interaction may be larger or smaller than for purely inertial interaction, depending on the decrease in the horizontal input motion and the increase in the rocking input motion. This approximation reflects the wave nature of the seismic excitation only in the transfer functions of the components of the foundation input motion. It would then be applicable not only to vertically propagating shear waves, but also to other types of incident waves using the appropriate values of Q_h and Q_r.

NUMERICAL RESULTS

Foundation input motions

Vertically incident shear waves give rise to a foundation input motion having horizontal and rocking components. The frequency-dependent values used herein are taken from tables reported by Mita and Luco[17] for the soil material dampings $\zeta_S = 0{\cdot}001$ and $\zeta_P = 0{\cdot}0005$. The real and imaginary parts and the amplitude of the ratios X_0/X_g and $L\Phi_0/X_g$ are depicted in Figure 4 for $D/L = 0{\cdot}5$ and $1{\cdot}5$. As can be seen, a considerable amount of rocking in the overall motion of the foundation may be induced, depending on the value of the well-known dimensionless frequency $a_0 = \omega L/c_s$. This frequency can also be written in terms of the dimensionless parameters D/L and τ_D referred to earlier as

$$a_0 = 2\pi\tau_D\frac{L}{D} \tag{25}$$

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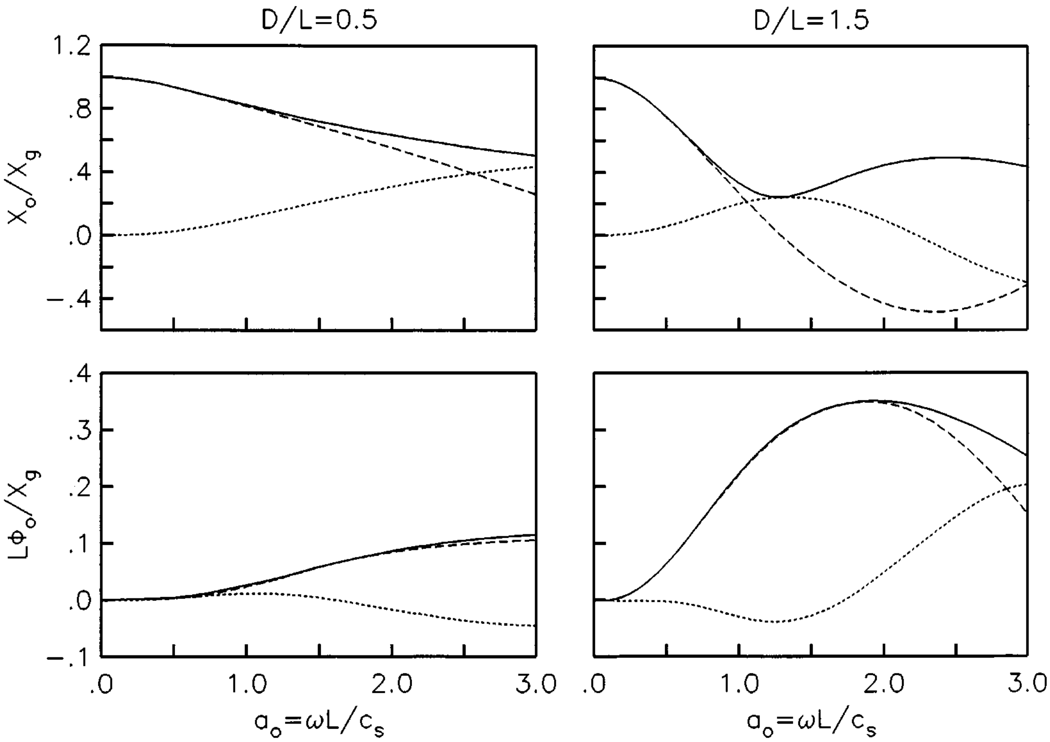

Figure 4. Real (dashed line) and imaginary (dotted line) parts and amplitude (solid line) of the translational and rocking components of the foundation input motion (adapted from Mita and Luco[17])

Consequently, for a specified value of D/L, the importance of the rocking induced by kinematic interaction depends on the value of τ_D. The latter may be interpreted as the ratio of the foundation embedment to the shear wave length $\lambda = c_s T_0$, where $T_0 = 2\pi/\omega$ is the period of the exciting motion. In general, the effects of foundation embedment on the translational and rocking input motions are most pronounced for the deeper foundation.

Effects of foundation embedment on system period and damping

Variations of the effective period and damping of interacting systems are shown in Figure 5 for $D/L = 0$, 0·5 and 1·5. The system period is normalized with respect to the fixed-base fundamental period. Two values of H/L are used: $H/L = 1$ corresponding to short squat structures, and $H/L = 3$ corresponding to tall slender structures. Some conclusions drawn from these results are similar to those reported previously for surface-supported structures.[3,4] The overall period of the system increases with increasing values of τ_H, and the overall damping decreases with increasing values of H/L. For short structures, the overall damping is always greater than the structural damping for the fixed-base condition, whereas for tall structures, interaction may increase or decrease the overall damping depending on the value of τ_H. Note further that the overall period of the system increases with increasing values of H/L, except for the deeper foundation. This fact was unexpected, even for solely inertial interaction.

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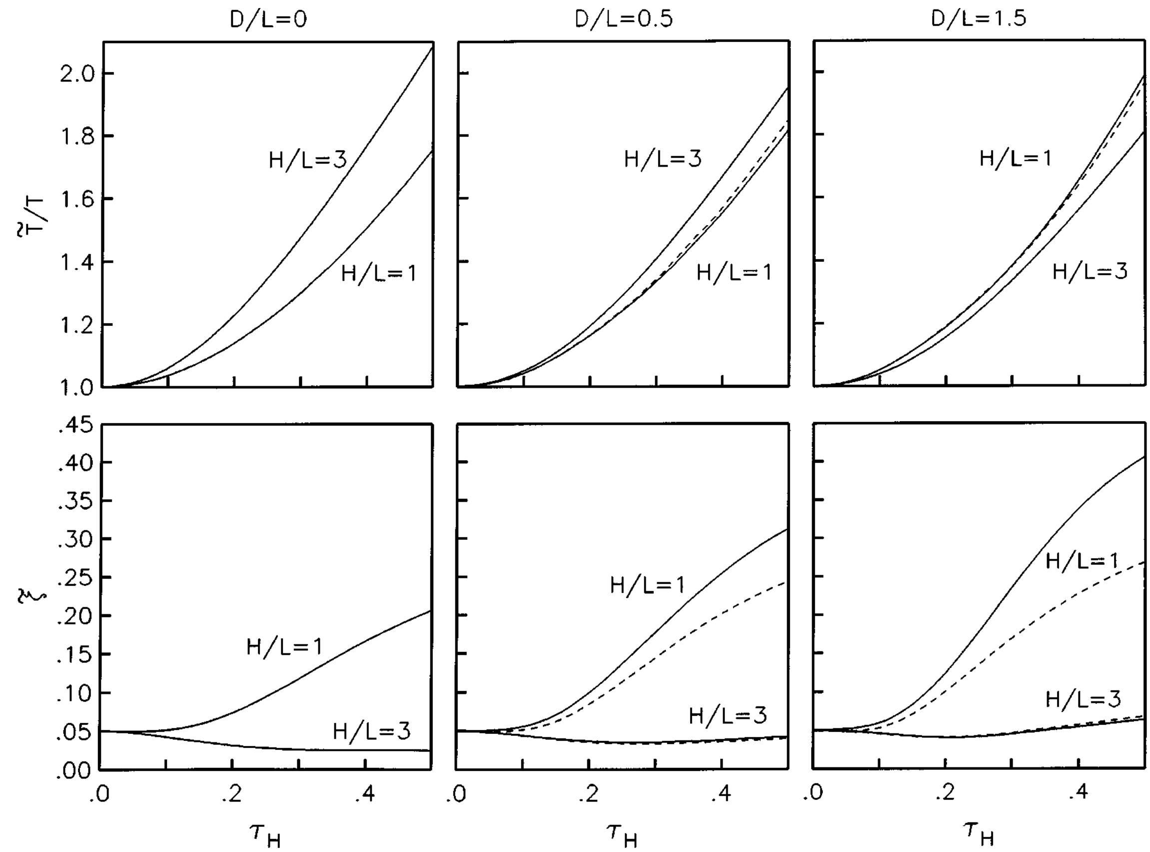

Figure 5. System periods and system dampings for total soil–structure interaction (solid line) and purely inertial interaction (dashed line)

It is clear from these results that the effects of foundation embedment are to decrease the system period for tall slender structures and to increase it for short squat structures, in addition to increasing the system damping for both structures. Results for total soil–structure interaction are also compared with those for purely inertial interaction. As anticipated, the system period is insensitive to kinematic interaction. Furthermore, for short structures, kinematic interaction substantially increases the system damping, while for tall structures, only insignificant reductions or increments are produced.

It is worth pointing out that the soil material damping used in calculations is very small. This factor may be essential because energy dissipation by hysteretic action in the soil reduces the effective period of the system, increases the effective damping and reduces the maximum structural deformation, as demonstrated earlier for surface-supported structures.[4] Generally, results may be quite sensitive to variations in the soil material damping.

The approximation for including kinematic interaction will now be verified. The exact system dampings given in Figure 5 for total soil–structure interaction are compared in Figure 6 with those obtained by the approximate solution. The degree of agreement between the two sets of results is indeed excellent. Although some differences are observed around $\tau_H = 0{\cdot}5$ for the short structure with the deeper foundation, they are of little practical importance. For a representative storey height of 3 m, the ratio H/T is approximately equal to 30 m/s for many types of buildings. Accordingly, the value of $\tau_H = 0{\cdot}5$ corresponds to very soft soils with $c_s = 60$ m/s.

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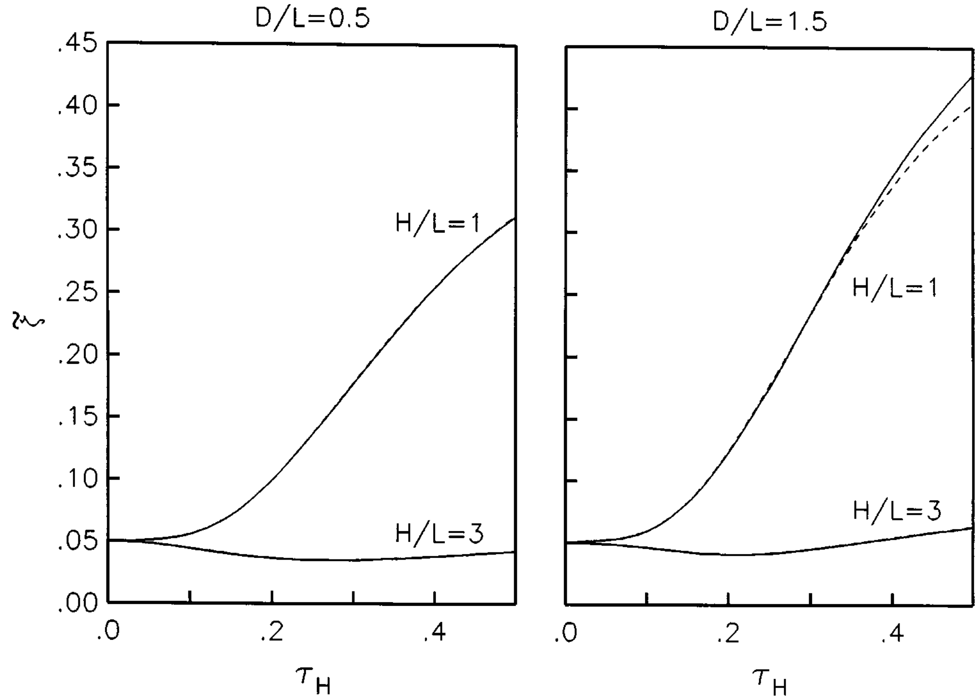

Figure 6. Comparison of the approximate (solid line) with the numerical solution (dashed line) for the system damping with both kinematic and inertial interaction

Effects of foundation embedment on system response

The following results correspond to the free-field excitation given by the NS component of the 1995 Manzanillo, Mexico, earthquake record. The acceleration, velocity and displacement traces of this record are depicted in Figure 7. The normalizing, characteristic period of the excitation is $T_0 = 0{\cdot}25$ s, which is interpreted as the period corresponding to the maximum spectral acceleration.

The transient response of the system was computed by application of discrete Fourier transforms, using the Fast Fourier Transform technique and taking due precautions to ensure that the aliasing error involved in their application was negligibly small. The details of the method of analysis are similar to those elsewhere[3,4] mentioned for surface-supported structures, except that the frequency dependency of both impedance functions and input motions for embedded foundations was properly taken into account.

In Figures 8 and 9 are given response spectra for interacting systems with $D/L = 0$, 0·5 and 1·5 and $H/L = 1$ and 3, considering two values of the foundation flexibility: $\tau_L = 0{\cdot}08$ and 0·2, respectively. The resulting values of the product $\tau_L T_0$ are the same as those used earlier[3,4] for surface-supported structures. In these plots, the ordinates represent the structural pseudo-acceleration normalized with respect to the acceleration of gravity, $\Omega^2 X_e/g$, and the abscissas represent the ratio T/T_0. Three sets of solutions are displayed: (1) for no soil–structure interaction, i.e. analysing the fixed-base structure under the free-field ground motion; (2) for inertial interaction only, i.e. analysing the interacting system under the free-field ground motion; and (3) for total soil–structure interaction, i.e. analysing the interacting system under the foundation input motion. The following conclusions are drawn from these results, some of which are similar to those reported previously for surface-supported structures.[3,4]

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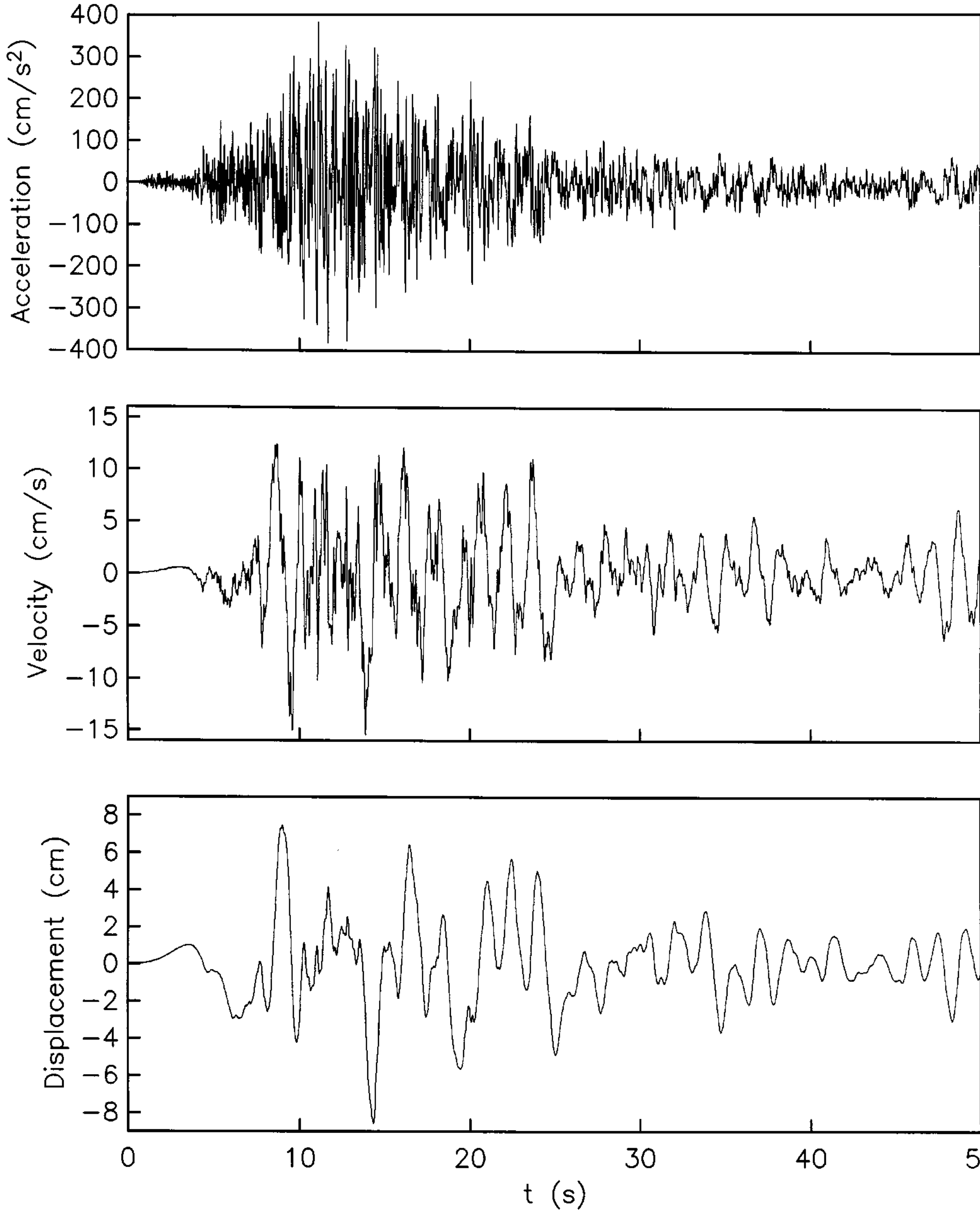

Figure 7. Acceleration, velocity and displacement time histories of the NS component of the 9 October 1995, Manzanillo earthquake

(1) As expected, the effects of soil–structure interaction are more pronounced for $\tau_L = 0{\cdot}2$ than for $\tau_L = 0{\cdot}08$. For a foundation with, say, $L = 20$ m, these values would correspond to soils with $c_s = 400$ and 1000 m/s, respectively.

(2) The interaction effects are significant in the spectral region $T/T_0 < 3$, corresponding to stiff short-period structures, and negligible in the spectral region $T/T_0 > 3$, corresponding to flexible long-period structures. Also, for interacting systems having the same foundation flexibility, the interaction effects are more important for tall slender structures than for short squat structures.

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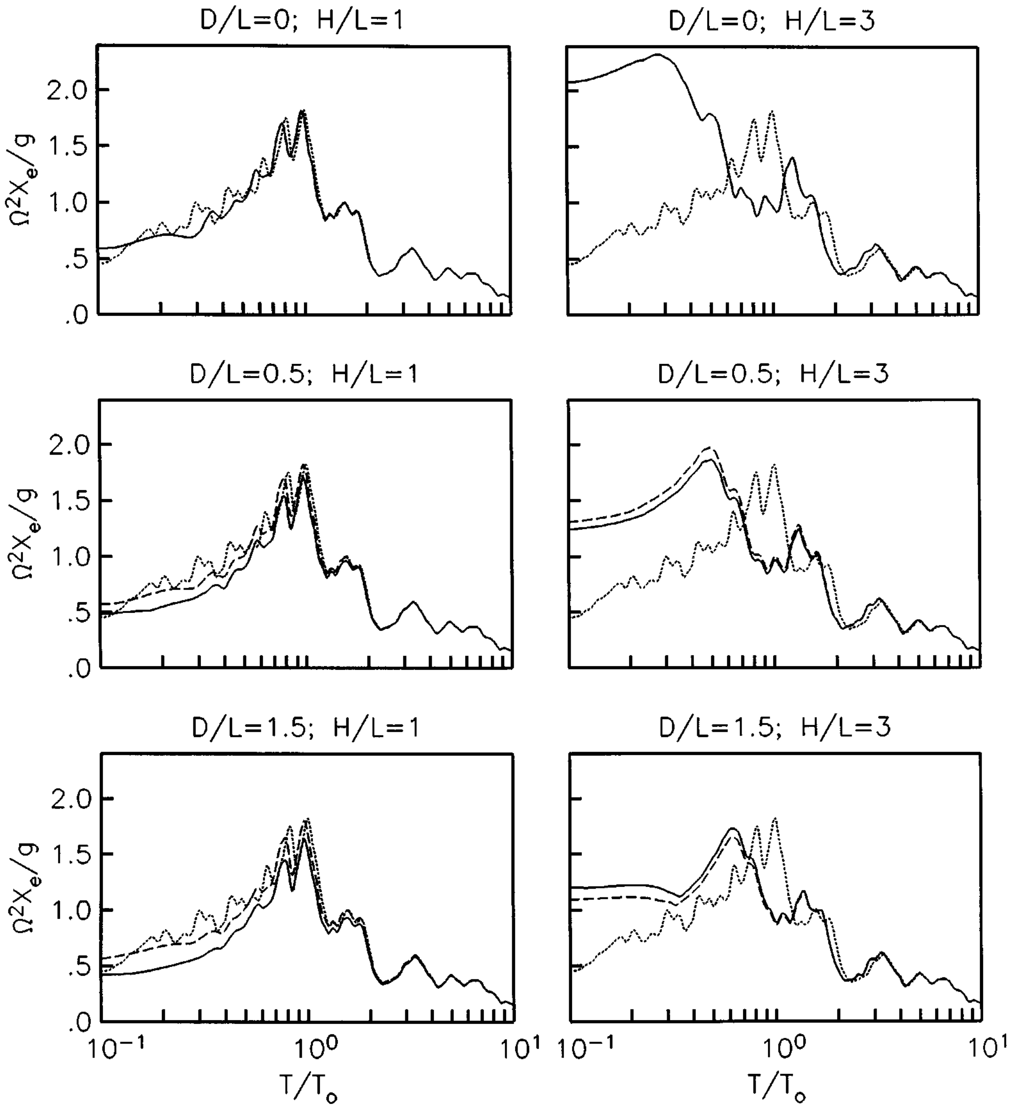

Figure 8. Deformation response spectra for interacting systems with $\tau_L = 0{\cdot}08$ subjected to the 1995 Manzanillo earthquake; no soil–structure interaction (dotted line), inertial interaction only (dashed line) and total soil–structure interaction (solid line)

(3) The effects of inertial interaction are, in general, more important than those of kinematic interaction. Irrespective of the foundation flexibility, kinematic interaction reduces the maximum response of short squat structures. The deeper the foundation, the greater the reduction becomes. In this case, the system damping increases significantly with increasing the embedment depth. For example, for $\tau_L = 0{\cdot}2$ and $H/L = 1$, a value of, say, $T/T_0 = 0{\cdot}8$ corresponds to $\tau_H = 0{\cdot}25$. Using this latter value in Figure 5 for the deeper foundation, the system damping is $\tilde{\zeta} = 0{\cdot}13$ for purely inertial interaction, and $\tilde{\zeta} = 0{\cdot}18$ for total soil–structure interaction. It is interesting to note that for tall slender structures, the interaction effects may result in reductions or increments in the system response, depending on the values of τ_L and T/T_0 involved. Generally, an increase in the system response is associated with unrealistic combinations of system parameters. For $\tau_L = 0{\cdot}08$, interaction significantly increases the maximum response of tall slender structures in the short-period

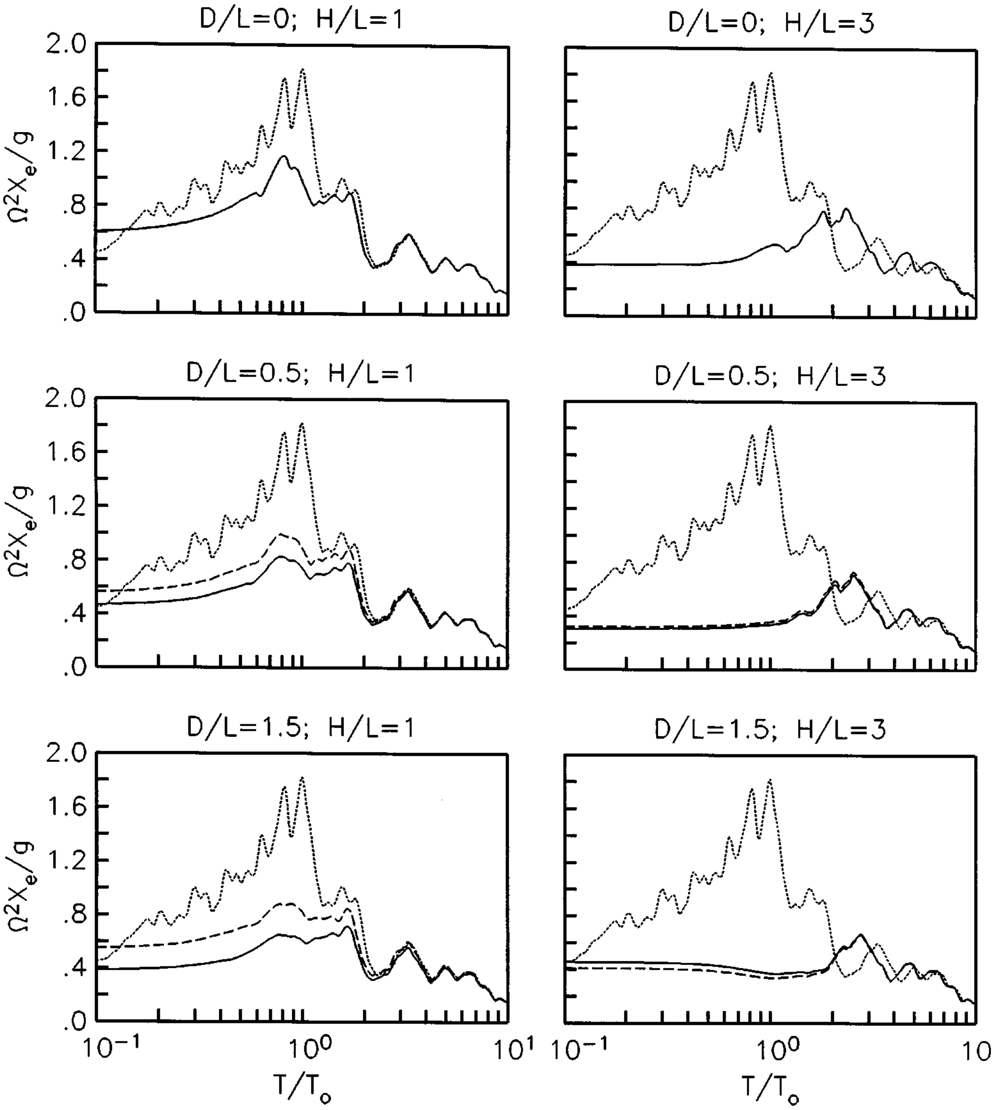

Figure 9. Deformation response spectra for interacting systems with $\tau_L = 0{\cdot}2$ subjected to the 1995 Manzanillo earthquake; no soil–structure interaction (dotted line), inertial interaction only (dashed line) and total soil–structure interaction (solid line)

spectral region. However, such structures normally fall in the long-period spectral region, for which the interaction effects are negligible.

(4) Independent of the foundation flexibility, the shapes of response spectra for surface-supported and embedded foundations are similar. The principal effects of foundation embedment are, in general, to accentuate the reductions in spectral ordinates. It should be noted that for tall slender structures, the peak spectral ordinate shifts considerably towards the left with respect to $T/T_0 = 1$ for $\tau_L = 0{\cdot}08$, and towards the right for $\tau_L = 0{\cdot}2$.

The concept of using free-field response spectra in combination with the effective period and damping of the system, considering both kinematic and inertial interaction, will now be verified. The response spectra for

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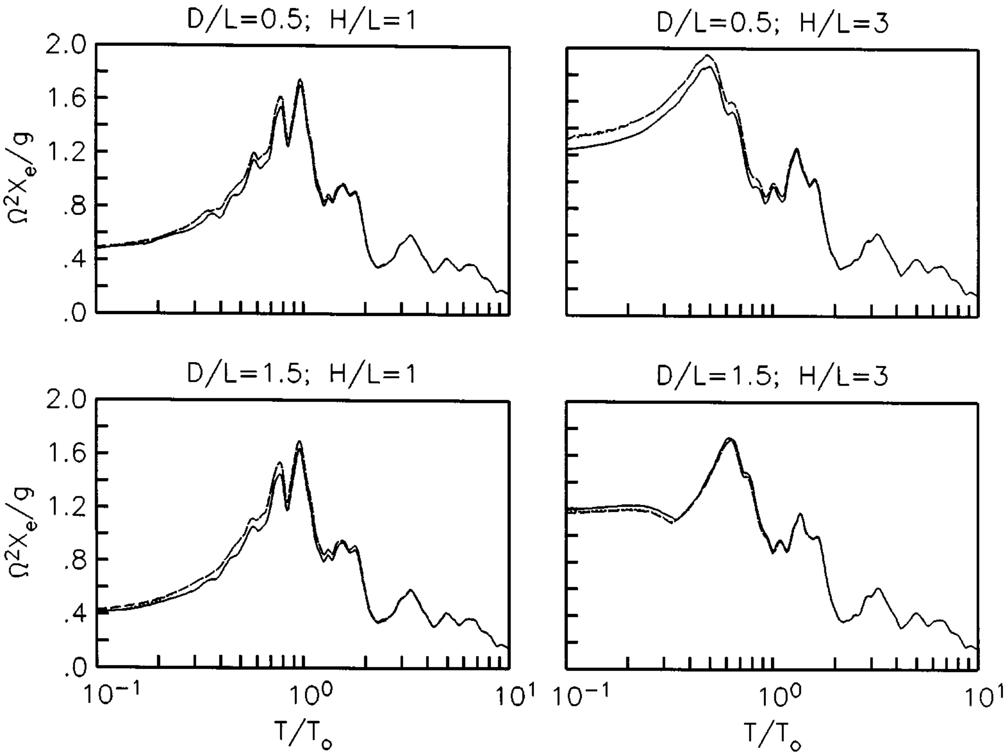

Figure 10. Comparison of the response spectra for the interacting system (solid line) with those for the replacement oscillator evaluated approximately (dashed line) and numerically (dotted line); total soil–structure interaction for $\tau_L = 0{\cdot}08$

the interacting system given in Figures 8 and 9 for total soil–structure interaction are compared in Figures 10 and 11 with those for the replacement oscillator, utilizing the system period and system damping determined numerically and approximately by the procedures presented herein. The degree of agreement among the three response spectra is very good for practical purposes. Although some differences are observed in the short-period spectral region, they have no practical consequences. This agreement confirms the reliability of the proposed approach for assessing the kinematic interaction effects due to the embedment depth.

CONCLUSIONS

Information and concepts to account for the effects of foundation embedment on the effective period and damping and the response of soil–structure systems have been presented. They were developed by reference to the replacement oscillator widely used in code interaction provisions. It has been shown that, provided the dynamic properties of the structure are modified for both inertial and kinematic interaction, the replacement oscillator approach supplies a reliable practical mean for estimating the peak structural response. The use of standard free-field response spectra applicable to the system period and system damping is then permitted. Moreover, the approximation developed to take kinematic interaction into account could be useful in improving seismic design provisions for building structures.

Results have been given for foundations embedded in an elastic half-space under vertically incident shear waves, indicating the influences of the more important parameters involved and the relative importance of

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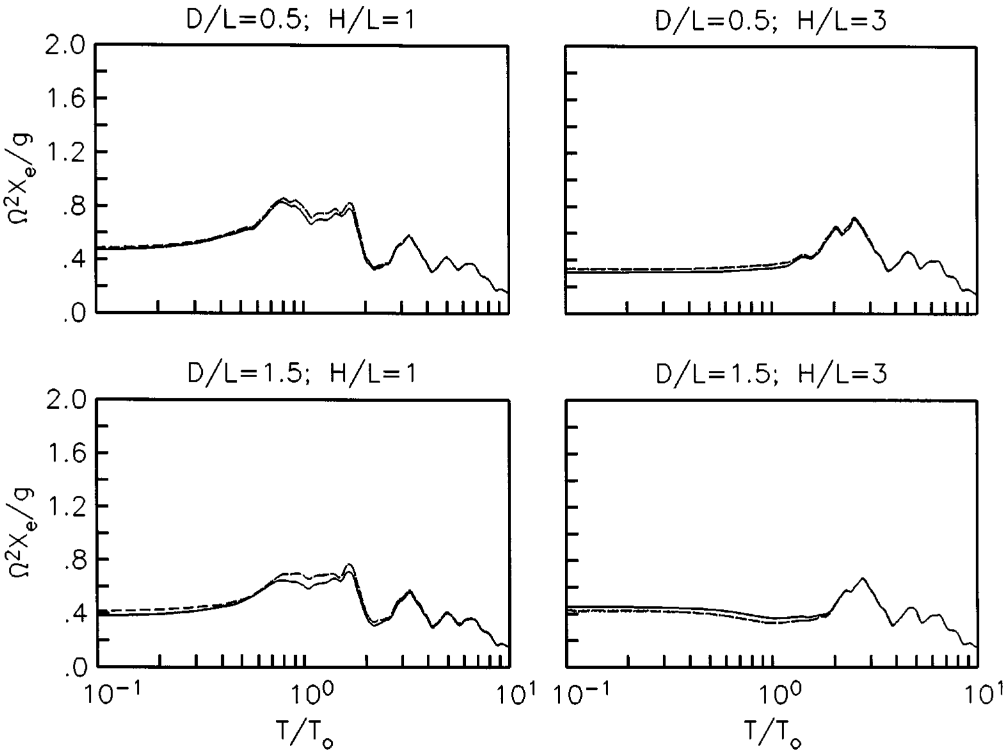

Figure 11. Comparison of the response spectra for the interacting system (solid line) with those for the replacement oscillator evaluated approximately (dashed line) and numerically (dotted line); total soil–structure interaction for $\tau_L = 0{\cdot}2$

the kinematic and inertial interaction. Nevertheless, the energy dissipated by the scattering and diffraction of the incident seismic waves from the foundation should be studied for other soil conditions, types of incident waves and soil material dampings, in order to better understand the damping capacity of the foundation for reducing the maximum deformation of the structure.

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REFERENCES

1. P. C. Jennings ad J. Bielak, 'Dynamics of building–soil interaction', *Bull. Seism. Soc. Am.* **63**, 9–48 (1973).
2. A. S. Veletsos, 'Dynamics of structure-foundation systems', in: W. J. Hall (ed.), *Structural and Geotechnical Mechanics*, Prentice-Hall, Englewood Cliffs, NJ, 1977, pp. 333–361.
3. A. S. Veletsos and J. W. Meek, 'Dynamic behaviour of building–foundation systems', *Earthquake Engng. Struct. Dyn.* **3**, 121–138 (1974).
4. A. S. Veletsos and V. V. D. Nair, 'Seismic interaction of structures on hysteretic foundations', *J. Struct. Div. ASCE* **101**, 109–129 (1975).
5. Applied Technology Council (ATC), 'Tentative provisions for the development of seismic regulations for buildings', *ATC-3-06 Amended*, California, 1984.
6. National Earthquake Hazard Reduction Program (NEHRP), 'Recommended provisions for the development of seismic regulations for new buildings', *FEMA-BSSC*, Washington, 1994.
7. A. S. Veletsos, 'Design concepts for dynamics of soil–structure interaction', in P. Gülkan and R. W. Clough (eds), *Developments in Dynamic Soil–Structure Interaction*, Kluwer Academic Publishers, The Netherlands, 1993, pp. 307–325.

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8. A. S. Veletsos and A. Prasad, 'Seismic interaction of structures and soils: stochastic approach', *J. Struct. Engng. ASCE* **115**, 935–956 (1989).
9. J. Bielak, 'Modal analysis for building–soil interaction', *J. Engng. Mech. Div. ASCE* **102**, 771–786 (1976).
10. J. E. Luco, 'Linear soil–structure interaction', *Report UCRL-15272*, *Lawrence Livermore National Laboratory*, Livermore, California, 1980.
11. J. E. Luco, 'Soil–structure interaction and identification of structural models', *Proc. ASCE Speciality Conf. in Civil Engineering and Nuclear Power*, Knoxville, Tennessee, 1980.
12. J. P. Wolf, *Dynamic Soil–Structure Interaction*, Prentice-Hall, Englewood Cliffs, NJ, 1985.
13. J. Avilés and L. E. Pérez-Rocha, 'Evaluation of interaction effects on the system period and the system damping due to foundation embedment and layer depth', *Soil Dyn. Earthquake Engng.* **15**, 11–27 (1996).
14. J. Bielak, 'Dynamic behavior of structures with embedded foundations', *Earthquake Engng. Struct. Dyn.* **3**, 259–274 (1975).
15. M. I. Todorovska and M. D. Trifunac, 'The system damping, the system frequency and the system response peak amplitudes during in-plane building–soil interaction', *Earthquake Engng. Struct. Dyn.* **21**, 127–144 (1992).
16. M. I. Todorovska, 'Effects of the depth of the embedment on the system response during building–soil interaction', *Soil Dyn. Earthquake Engng.* **11**, 111–123 (1992).
17. A. Mita and J. E. Luco, 'Impedance functions and input motions for embedded square foundations', *J Geotech. Engng. ASCE* **115**, 491–503 (1989).

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